Illinois Department of Transportation

Memorandum

To:	ALL BRIDGE DESIGNERS	03.4
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Subject:	BRIDGES AND STRUCTURES	
Date:	July 11, 2003	

This memorandum is the second in a series detailing the Department's policies and procedures for implementation of the AASHTO LRFD Bridge Design Specifications by October 1, 2007. The previous ABD memo 02.3 addressed LRFD deck slab and slab bridge designs. This memorandum details an IDOT developed standard splice design procedure for rolled beams as well as policies concerning the design of elastomeric bearings and miscellaneous structural steel design for typical bridges in Illinois designed according to LRFD provisions.

Standard Splices

A procedure for structural steel splice design for rolled beams was developed which satisfies both the 1998 AASHTO LRFD Specifications through the 2003 Interims as well as the 2002 LFD Standard Specifications. A procedural description, example splice design calculations, and standard splice designs using both Grade 36 and 50 steels for 39 AISC rolled beam shapes are all included in this memo. Clarification of broadly outlined methods in LRFD and LFD for splice design is one of several purposes for providing a detailed IDOT methodology description as well as a set of example calculations. The standard splice designs shall be applicable to all LRFD and LFD design plans prepared from TSL plans approved after September 1, 2003 and may be used, when feasible, for bridges where the final design plans have not been completed.

Elastomeric Bearings

The procedure for the design of elastomeric bearings under the LRFD Bridge Design Specifications remains unchanged from the current methods detailed in the IDOT Bridge Manual. Only the design loads for bearings using the LRFD Code will be somewhat different from those of LFD due to the new HL-93 design loading and load distribution factors detailed in Sections 3 and 4 of the LRFD Provisions. LRFD factored service reactions from load case Service I given in Table 3.4.1-1 shall be the governing load case. The end shear/reaction amplification for skewed bridges given in LRFD Article 4.6.2.2.3c is not applicable for interior and exterior beam bearing designs at piers. The factored resistance of structural steel in bearing assemblies shall be calculated as currently stipulated in the IDOT Bridge Manual.

Miscellaneous Steel Design

For composite beam design, the in-place requirements of Article 3.3.7 of the Bridge Manual shall apply to all LRFD designs. Longitudinal reinforcement over piers shall not be used in the calculation of the section properties as specified in AASHTO LRFD Articles 6.10.3.1.1c and 6.6.1.2.1, and the longitudinal steel requirements of Article 6.10.3.7 are not applicable.

Horizontally Curved Girders

The 2003 AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges shall be applicable to all designs where the final plans have not been completed with the exception of Article 1.1 requirement for composite girders to be designed composite along the entire length of the girder. Composite horizontally curved steel girders shall not be designed composite along the entire length of the girder as specified. The current policy of limiting composite design to positive moment regions for non-curved girders shall also be applicable to horizontally curved girders. Additionally, as permitted by Article 11.2 of this guide specification, the oversize and slotted holes provisions stated in Articles 3.3.19 and 3.3.20 of the Bridge Manual shall be applicable to curved girder bridges designed with these specifications.

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Design Procedure

LRFD and LFD Symmetrical Splices for Typical Illinois Bridges
W-Shapes

General

Standard symmetrical splice designs for Wide Flange Sections were developed (Figs. 1 and 2, and Tables 1 and 2) according to applicable sections, sub-sections and referenced sections of LRFD 6.13.1 & .2 (1998 Edition through 2003 Interims) and LFD 10.18.1 & .2 (2002 Edition). The standardized details are appropriate when splices are located at or near dead load contraflexure points (LRFD 6.13.6.1.4a, LFD 10.18.2.2.1), i.e. $\pm$ 5% of span length, and are valid for superstructures designed according to AASHTO LRFD and LFD. The two code's specific procedures differ at times. For this reason, the more conservative specification governed that aspect of standard splice design. Splices for other compact W-Shape sections meeting the general requirements above can be designed using the procedure outlined below.

Materials

Structural Steel

Splices for W-shape beams using both Grade 36 and Grade 50 Steels were developed. The following basic material properties apply:

AASHTO M270 Grade 36	$F_y = 36$ ksi	$F_u = 58$ ksi
AASHTO M270 Grade 50	$F_y = 50$ ksi	$F_u = 65$ ksi
AASHTO M270 Grade 50W	$F_y = 50$ ksi	$F_u = 65$ ksi *

* Reduced from 70 ksi

Bolts

All bolts in standard splice designs are AASHTO M 164 (ASTM A 325) 7/8"ϕ with Class A Contact Surfaces and Standard Sized Holes.

Design Forces

General

Splices are designed for not less than 75% of the capacity (resistance/design strength) of the member (LRFD 6.13.1 and LFD 10.18.1.1). Generally, the average of the member capacity and actual applied factored design load shall also be investigated but will not govern for typical splices located near points of contraflexure. Computed design forces are considered factored ultimate loads even though they are not explicitly "factored" in the conventional sense according to LFD and LRFD.

Flange Plates

The design stress for each specification is,

$$F_{cf} = 0.75\alpha\phi_f F_{yf} \qquad \text{(LRFD 6.13.6.1.4c)}$$

Where :

$$\alpha = 1.0, \phi_f = 1.0, F_{yf} = 36 \text{ or } 50 \text{ ksi}$$

and,

$$F_{cu} = 0.75\alpha F_{yf} \qquad \text{(LFD 10.18.2.2)}$$

Where :

$$\alpha = 1.0, F_{yf} = 36 \text{ or } 50 \text{ ksi}$$

Other cases (compression, "non-controlling" flange plates for asymmetrical connections) stated in the above sections of both specifications do not apply for standard splices.

Design stress is multiplied by an Effective Area (A_e) for determination of final factored flange splice plate design force. In LRFD, Effective Area is calculated from,

$$A_e = A_n + \beta A_g \le A_g \qquad \text{(LRFD 6.10.3.6)}$$

Where :

A_n = Net Area of Flange (LRFD 6.8.3)

A_g = Gross Area of Flange

$$\beta = \left(\frac{A_n}{A_g}\right)\left(\left[\frac{\phi_u F_u}{\phi_y F_{yf}}\right] - 1\right)$$

ϕ_u = resistance factor for fracture in tension (LRFD 6.5.4.2)

ϕ_y = resistance factor for yield in tension (LRFD 6.5.4.2)

F_u = 58 or 65 ksi

And for LFD it is,

$$A_e = W_n t + \beta A_g \le A_g \qquad \text{(LFD 10.18.2.2.4)}$$

Where :

W_n = Least net width of the flange for design force determination, or that of splice plate for strength check (LFD 10.16.14)

t = flange or splice plate thickness

A_g = Gross Area of Flange

$\beta = 0.15$

The larger force calculated from the two code procedures governs for an individual standard splice.

Web Plates

Section 6.13.6.1.4b of LRFD and LFD 10.18.2.3 give the methodology for calculation of web plate factored design shear. Both codes are equivalent with only differing notations employed. In LRFD terminology, the design force is computed as,

$$\begin{aligned}
&\text{If } V_u < 0.5V_r \text{ then :}\\
&V_{uw} = 1.5V_u \qquad\qquad \text{(Case 1)}\\
&\text{Otherwise :}\\
&V_{uw} = \frac{V_u + V_r}{2} \qquad\qquad \text{(Case 2)}\\
&\text{Where :}\\
&V_u = \text{shear due to the factored loading at the point of splice}\\
&V_r = \text{factored shear resistance at the point of splice}\\
&\quad \text{(LRFD 6.10.7.1, LFD 10.48.8.1. Note : Both codes produce}\\
&\quad \text{the same shear resistance. The units, lbs vs. kips, are different}\\
&\quad \text{and } \phi_v = 1.0 \text{ in LRFD)}
\end{aligned}$$

Substituting $0.5V_r$ in the right hand term of Case 1 produces the "75% rule" from above. Replacing V_u with $0.5V_r$ in Case 2 produces the 75% rule as well. In simple terms, this is LRFD and LFD's way of stating the 75% and "average" rules non-explicitly.

The considered design moment for the web plates is found by multiplying the design shear from above by the distance from the centroid of the connection as a whole to the centroid of a web bolt group on one member being connected.

Slip

The design slip forces for bolts are service factored loads, not ultimate factored loads (LRFD 6.13.2.2, 6.13.6.1.4b, 6.13.6.1.4c; and LFD 10.18.2.2.2, 10.18.2.3.5). In LRFD, the calculated forces from Service Load Case II are appropriate while in LFD the case to be used is commonly referred to as Overload. The smallest ultimate load factor in either LRFD or LFD is 1.25. Multiplying the inverse of 1.25 by the ultimate factored computed loads above for the flange and web plates provides a conservative estimate of the design slip force for standardized splices.

Bolt Shear and Bearing

The design tension divided by the number of bolts on one side of the connection is the design force per bolt or bearing surface for both LRFD and LFD in the flanges.

The design bolt shear and bearing forces in the web plates is arrived at by combining the effects derived from the design shear and moment in the web connection. The design force per bolt or bearing surface due to design shear is computed as above for the flange plates. The added design force due to moment is computed using the design shear only and a polar moment of inertia formulation as for an eccentric shear connection.

<u>Design Resistances</u>

Flange Plates

In LRFD, tension on the gross section (6.13.5.2) and fracture on the net section (6.13.5.2) must be checked as potential failure modes for the flange splice plates. These are conventional checks identical to those of simple tension members. Typically, splices are not susceptible to block shear. Consequently, LRFD Article 6.13.4 is not applicable to standard splice designs. Tension on the Effective Area as defined above need only be checked in LFD (10.18.2.2).

Web Plates

In LRFD, shear on the gross section ($\phi_v 0.58 F_y A_g$) (Art. 6.13.5.3) and flexure on the gross section (Mc/I_g less than $\phi_f F_y$) (Art. 6.13.6.1.4b) must be checked for the web plates. These are conventional checks identical to those for simple flexural members. Typically, splices are not susceptible to block shear. Consequently, LRFD Article 6.13.4 is not applicable to standard splice designs. Similar shear and flexure checks on the gross section are required in LFD (10.18.2.3).

Bolt Shear

The factored resistance of bolts subjected to shear using LRFD is given in 6.13.2.7,

$$\phi_s R_n = \phi_s \; 0.38\, A_b\, F_{ub}\, N_s = 21.94\, N_s \;{}^{\text{kips}}\!/_{\text{bolt}}$$

Where :

$\phi_s = 0.8$

$A_b = 0.6013\ \text{in}^2$

$F_{ub} = 120\ \text{ksi}$

$N_s =$ Number of Shear Planes

For LFD, the applicable article is 10.56.1.3.2,

$$\phi R = \phi F A_b N_s = 21.05\, N_s \;{}^{\text{kips}}\!/_{\text{bolt}}$$

Where :

$\phi F = 35\ \text{ksi}$

$A_b = 0.6013\ \text{in}^2$

$N_s =$ Number of Shear Planes

The lesser value (21.05) was used for resistance of the bolts.

Bolt Bearing

Bearing resistance in LRFD is described in section 6.13.2.9,

$$\phi_{bb} R_n = \phi_{bb} 1.2 L_c t F_u = 55.68t \text{ or } 62.40t \ {}^{kips}\!/_{bolt}$$

Where :

$\phi_{bb} = 0.8$

$L_c = 1"$

$t = \text{bearing surface thickness}$

$F_u = 58 \text{ or } 65 \text{ ksi}$

In LFD (10.56.1.3.2) it is,

$$\phi R = 0.9 L_c t F_u = 52.2t \text{ or } 58.5t \ {}^{kips}\!/_{bolt}$$

Where :

$L_c = 1"$

$t = \text{bearing surface thickness}$

$F_u = 58 \text{ or } 65 \text{ ksi}$

The lesser values (LFD's) were used for resistance to bearing forces.

Slip

All bolts in standard splice designs are AASHTO M 164 (ASTM A 325) 7/8" ϕ with Class A Contact Surfaces and Standard Sized Holes. The factored resistance per bolt in LRFD (6.13.2.8) given by,

$$\phi R_n = \phi K_h K_s N_s P_t = 12.87 N_s \ {}^{kips}\!/_{bolt}$$

Where :

$\phi = 1.0$

$K_h = 1.0$

$K_s = 0.33$

$N_s = \text{Number of Slip Planes}$

$P_t = \text{Minimum Required Bolt Tension, 39 kips}$

For LFD (10.57.3), the factored resistance per bolt is,

$$\phi R_s = \phi F_s A_b N_s = 12.63 N_s \ {}^{kips}\!/_{bolt}$$

Where :

$\phi F_s = 21 \text{ ksi}$

$N_s = \text{Number of Slip Planes}$

$A_b = 0.6013 \text{ in}^2$

The lesser value (12.63) was used for slip resistance of the bolts.

Splice Geometry

Flange

Double flange plates were employed when geometrically feasible according to accepted standard center-to-center bolt and edge distances. The minimum pitch and gage employed was 1.5" when bolt patterns were staggered. Staggered bolt patterns were geometrically optimized to produce the shortest flange splice plate lengths.

Web

Web bolt patterns did not employ stagger. The maximum number of bolts in a row were geometrically fitted as recommended by the specifications. The shortened available web depth according to new rolling practices was taken into account. At times, a few extra bolts were required to meet the minimum of two rows in each member being connected.

Standard Splice Example - W33 x 201

Materials

Structural Steel

AASHTO M270 Grade 50 $F_y = 50\ \text{ksi}$ $F_u = 65\ \text{ksi}$

Bolts

AASHTO M 164 (ASTM A 325) 7/8" ϕ with Class A Contact Surfaces and Standard Sized Holes.

Design Forces

Flange Plates

LRFD Design Stress

$$F_{cf} = 0.75\alpha\phi_f F_{yf} = 37.5\ \text{ksi}$$

Where :

$$\alpha = 1.0,\ \phi_f = 1.0,\ F_{yf} = 50\ \text{ksi}$$

LFD Design Stress,

$$F_{cu} = 0.75\alpha F_{yf} = 37.5\ \text{ksi}$$

Where :

$$\alpha = 1.0,\ F_{yf} = 50\ \text{ksi}$$

LRFD Effective Area (2 bolts w/o stagger, 4 bolts w/ stagger),

$$A_e = A_n + \beta A_g \le A_g$$

$$A_n = (15.7 \times 1.15) - \left[\left(\tfrac{7}{8} + \tfrac{1}{8}\right) \times 2 \times 1.15\right] = 15.76\ \text{in}^2\ \text{(w/o stagger)}$$

$$A_n = (15.7 \times 1.15) - \left[\left(\tfrac{7}{8} + \tfrac{1}{8}\right) \times 4 \times 1.15\right] + \left(2 \times \frac{1.75^2}{4 \times 2.5} \times 1.15\right) = 14.16\ \text{in}^2\ \text{(w/ stagger)}$$

$$\therefore \text{Use } 14.16\ \text{in}^2$$

$$A_g = 15.7 \times 1.15 = 18.06\ \text{in}^2$$

$$\beta = \left(\frac{A_n}{A_g}\right)\left(\left[\frac{\phi_u F_u}{\phi_y F_{yf}}\right] - 1\right) = \left(\frac{14.16}{18.06}\right)\left(\frac{0.8 \times 65}{0.95 \times 50} - 1\right) = 0.074$$

$$\phi_u = 0.80,\ \phi_y = 0.95$$

$$A_e = 14.16 + (0.074 \times 18.06) = 15.50 < 18.06\ \therefore \text{Use } 15.50\ \text{in}^2$$

LFD Effective Area (2 bolts w/o stagger, 4 bolts w/ stagger),

$$A_e = W_n t + \beta A_g \le A_g$$
$$W_n = 15.7 - \left[\left(7/8 + 1/8\right) \times 2\right] = 13.7 \text{ in (w/o stagger)}$$
$$W_n = 15.7 - \left[\left(7/8 + 1/8\right) \times 4\right] + \left(2 \times \frac{1.75^2}{4 \times 2.5}\right) = 12.31 \text{ in (w/ stagger)}$$
$$\therefore \text{Use } 12.31 \text{ in}$$
$$t = 1.15 \text{ in},\ A_g = 18.06 \text{ in}^2,\ \beta = 0.15$$
$$A_e = (12.31 \times 1.15) + (0.15 \times 18.06) = 16.87 < 18.06 \therefore \text{Use } 16.87 \text{ in}^2$$

LFD governs the Flange Splice Plate Design Force :

$$P_{\text{Design Flange (Ultimate)}} = 16.87 \text{ in}^2 \times 37.5 \text{ ksi} = 633 \text{ kips}$$
$$P_{\text{Design Flange (Service)}} = \frac{633 \text{ kips}}{1.25} = 506 \text{ kips}$$

Web Plates

The design shear force for LRFD and LFD is,

$$V_{uw} = 0.58 \times 50 \times 31.4 \times 0.715 = 651 \text{ kips } (C = 1.0)$$
$$V_{\text{Design Web (Ultimate)}} = 0.75 \times V_{uw} = 0.75 \times 651 = 488 \text{ kips}$$
$$V_{\text{Design Web (Service)}} = \frac{488 \text{ kips}}{1.25} = 390 \text{ kips}$$

The design moment for LRFD and LFD is,

Bolt pattern $\rightarrow$ 4 vertical lines of bolts with 9 bolts per line at 3" spacing.

Moment arm $= 2" + 3" + 1\frac{1}{2}" = 6\frac{1}{2}"$ (see Fig. 2)

$$M_{\text{Design Web (Ultimate)}} = 6\tfrac{1}{2}" \times 488 \text{ kips} = 3172 \text{ kip-in}$$
$$M_{\text{Design Web (Service)}} = \frac{3172 \text{ kip-in}}{1.25} = 2538 \text{ kip-in}$$

<u>Trial Design</u>

Flange Plates/Bolts

No of bolts based on slip,

$$N_b = \frac{506\text{ kips}}{(21 \times 0.6013)^{\text{kips}}/_{\text{bolt}} \times 2\text{ slip planes}} = 20.03\text{ bolts} \therefore \text{Try 11 staggered rows with 4 lines}$$

parallel to line of force $= 22$ bolts/flange

Plate Dimensions (Width and Thickness),

Top Flange Width $= 15.7$ in $\rightarrow \therefore$ Try an outside plate width of $15\ \tfrac{1}{2}$"

Try an inside plate width :

$$\frac{15\ \tfrac{1}{2}"}{2} - \frac{0.715"}{2} - \tfrac{1}{2}"-1\tfrac{3}{8}"-1\tfrac{1}{2}"+1\tfrac{1}{2}" = 5.52"$$

$$\left(\begin{array}{l} t_w = 0.715", \text{Assumed web plate } t = \tfrac{1}{2}", \\ \text{Horizontal clear} = 1\tfrac{3}{8}", \text{Edge distance} = 1\tfrac{1}{2}", \\ \text{Maximum bolt shank extension from web} = 1\tfrac{1}{2}" \end{array}\right)$$

or

$$\frac{15\ \tfrac{1}{2}"}{2} - 1\tfrac{3}{16}" = 6\tfrac{9}{16}"\ \left(k_1 = 1\tfrac{3}{16}"\right)$$

$$\therefore 5\ \tfrac{1}{2}"$$

Top Flange Thickness $= 1.15$ in $\rightarrow \therefore$ Try an outside plate thickness of $\tfrac{5}{8}$"

Try an inside plate thickness of $\tfrac{11}{16}$"

Web Plates/Bolts

Assumed bolt pattern,

4 vertical lines of bolts with 9 bolts per line at 3" spacing.

$$I_x = 4\text{ bolts per horizontal row} \times \sum_{i=1}^{n\text{ rows}} y_i^2 = 2160\text{ in}^2$$

$$I_y = 9\text{ bolts per verical line} \times \sum_{i=1}^{n\text{ lines}} x_i^2 = 405\text{ in}^2$$

$$I_p = 2160 + 405 = 2565\text{ in}^2$$

Max. Service Force "Extreme Fiber" Bolt

$$V_{force/bolt} \text{ per shear plane } = \frac{\frac{390 \text{ kips}}{36 \text{ bolts}}}{2 \text{ planes}} = 5.4 \text{ kips from pure shear} \downarrow$$

$$V_{force/bolt} \text{ per shear plane} = \frac{\frac{2538 \text{ kip-in} \times 4\ 1/2 \text{ in}}{2565 \text{ in}^2}}{2 \text{ planes}} = 2.2 \text{ kips max. shear from moment} \downarrow$$

$$V_{force/bolt} \text{ per shear plane} = \frac{\frac{2538 \text{ kip-in} \times 12 \text{ in}}{2565 \text{ in}^2}}{2 \text{ planes}} = 5.9 \text{ kips max. shear from moment} \rightarrow$$

$$V_{Total\ force/bolt} \text{ per shear plane } = \sqrt{(5.4 + 2.2)^2 + 5.9^2} = 9.62 \text{ kips max.} < 0.6013 \times 21 \text{ kips allowable for slip } \therefore \text{OK}$$

Plate Dimensions (Height and Thickness),

$\text{Member Depth} = 33.7 \text{ in.} \rightarrow \therefore$ Try a plate height of of :

$$33.7\text{"} - 2 \times \left(1.15\text{"} + 1\tfrac{1}{16}\text{"} + 1\tfrac{3}{8}\text{"} - 1\tfrac{1}{2} + 1\tfrac{1}{2}\right) = 27.28\text{"}$$

$$\left(\begin{array}{l} t_f = 1.15\text{", Bottom flange splice } t = 11/16\text{", Vertical} \\ \text{clearance} = 1\tfrac{3}{8}\text{", Min. edge distance} = 1\tfrac{1}{2}\text{",} \\ \text{Maximum bolt shank extension from flange} = 1\tfrac{1}{2}\text{"} \end{array} \right)$$

or

$$33.7\text{"} - 2 \times 2\text{"} = 29.7\text{"} \quad (k = 2\text{"})$$

$\therefore$ Use $27\ 1/4$"

$\text{Web Thickness} = 0.715 \text{ in} \rightarrow \therefore$ Try a plate thickness of $1/2$"

Ultimate Strength Checks

Flange Plates/Bolts

Bolt Shear,

$$P_{Design\ force/bolt} \text{ per shear plane } = \frac{633 \text{ kips}}{2 \text{ planes} \times 22 \text{ bolts}} = 14.4 \text{ kips}$$

$$P_{Allowable\ force/bolt} \text{ per shear plane } = 35 \times 0.6013 = 21.05 \text{ kips } \therefore \text{OK}$$

Bolt Bearing,

$$P_{Design} \text{ force per bearing surface } = \frac{633 \text{ kips}}{22 \text{ surfaces}} = 28.8 \text{ kips}$$

$$P_{Allowable} \text{ force per bearing surface } = 0.9 \times L_c \times F_u \times 1.15\text{"} \quad (L_c = 1\text{"}, F_u = 65 \text{ ksi})$$

$$58.5 \times 1.15\text{"} = 67.3 \text{ kips } \therefore \text{OK}$$

LRFD Tension on Gross Section,

$$P_{Design\ force} = 633\ \text{kips}$$

$$P_{Allowable\ force} = \phi_y A_g F_y = 0.95 \times \left[\left(5/8 \times 15\ 1/2\right) + 2 \times \left(5\ 1/2 \times 11/16\right)\right] \times 50 = 819\ \text{kips}\ \therefore \text{OK}$$

LRFD Tension on Net Section,

$$P_{Design\ force} = 633\ \text{kips}$$

$$P_{Allowable\ force} = \phi_u A_n F_u$$

$$\text{Outside plate } A_n = 5/8 \times \left[15\ 1/2 - 2 \times \left(7/8 + 1/8\right)\right] = 8.44\ \text{in}^2\ \text{(w/o stagger)}$$

$$\text{Outside plate } A_n = 5/8 \times \left\{\left[15\ 1/2 - 4 \times \left(7/8 + 1/8\right)\right] + \left(2 \times \frac{1.75^2}{4 \times 2.5}\right)\right\}$$

$$= 7.57\ \text{in}^2\ \text{(w/ stagger)}$$

$$\text{Inside Plates } A_n = 2 \times \left\{11/16 \times \left[5\ 1/2 - 1 \times \left(7/8 + 1/8\right)\right]\right\} = 6.19\ \text{in}^2\ \text{(w/o stagger)}$$

$$\text{Inside Plates } A_n = 2 \times \left\langle 11/16 \times \left\{\left[5\ 1/2 - 2 \times \left(7/8 + 1/8\right)\right] + \left(1 \times \frac{1.75^2}{4 \times 2.5}\right)\right\}\right\rangle$$

$$= 5.23\ \text{in}^2\ \text{(w/ stagger)}$$

$$\therefore A_n = 7.57 + 5.23 = 12.80\ \text{in}^2$$

$$P_{Allowable\ force} = \phi_u A_n F_u = 0.80 \times 12.80 \times 65 = 666\ \text{kips} > 633\ \text{kips}\ \therefore \text{OK}$$

LFD Tension on Effective Area,

$$P_{Design\ force} = 633\ \text{kips}$$

$$A_e = W_n t + \beta A_g \leq A_g$$

$$W_{n\ outside\ plate} = 15\ 1/2 - \left[\left(7/8 + 1/8\right) \times 2\right] = 13.5\ \text{in (w/o stagger)}$$

$$W_{n\ outside\ plate} = 15\ 1/2 - \left[\left(7/8 + 1/8\right) \times 4\right] + \left(2 \times \frac{1.75^2}{4 \times 2.5}\right) = 12.11\ \text{in (w/ stagger)}$$

$$t_{outside\ plate} = 5/8\ \text{in},\ A_{g\ top\ plate} = 9.69\ \text{in}^2,\ \beta = 0.15$$

$$A_{e\ outside\ plate} = \left(12.11 \times 5/8\right) + \left(0.15 \times 9.69\right) = 9.02 < 9.69 \therefore \text{Use } 9.02\ \text{in}^2$$

$$W_{n\ inside\ plates} = \left(5\ 1/2 \times 2\right) - \left[\left(7/8 + 1/8\right) \times 2\right] = 9.00\ \text{in (w/o stagger)}$$

$$W_{n\ inside\ plates} = \left(5\ 1/2 \times 2\right) - \left[\left(7/8 + 1/8\right) \times 4\right] + \left(2 \times \frac{1.75^2}{4 \times 2.5}\right) = 7.61\ \text{in (w/ stagger)}$$

$t_{\text{inside plates}} = {}^{11}\!/_{16}\ \text{in},\ A_{\text{g bot plates}} = 7.56\ \text{in}^2,\ \beta = 0.15$

$A_{\text{e inside plates}} = \left(7.61 \times {}^{11}\!/_{16}\right) + (0.15 \times 7.56) = 6.37 < 7.56 \therefore \text{Use } 6.37\ \text{in}^2$

$A_{\text{e total}} = 9.02 + 6.37 = 15.39\ \text{in}^2$

$P_{\text{Allowable force}} = A_{\text{e total}} F_y = 15.39 \times 50 = 770\ \text{kips} > 633\ \text{kips}\ \therefore \text{OK}$

Web Plates/Bolts

Max. Bolt Shear,

$$V_{\text{force/bolt per shear plane}} = \frac{\dfrac{488\ \text{kips}}{36\ \text{bolts}}}{2\ \text{planes}} = 6.8\ \text{kips from pure shear} \downarrow$$

$$V_{\text{force/bolt per shear plane}} = \frac{\dfrac{3172\ \text{kip-in} \times 4.5\ \text{in}}{2565\ \text{in}^2}}{2\ \text{planes}} = 2.8\ \text{kips max. shear from moment} \downarrow$$

$$V_{\text{force/bolt per shear plane}} = \frac{\dfrac{3172\ \text{kip-in} \times 12\ \text{in}}{2565\ \text{in}^2}}{2\ \text{planes}} = 7.4\ \text{kips max. shear from moment} \rightarrow$$

$$V_{\text{Design force/bolt per shear plane}} = \sqrt{(6.8 + 2.8)^2 + 7.4^2} = 12.1\ \text{kips max.}$$

$$V_{\text{Allowable force/bolt per shear plane}} = 35 \times 0.6013 = 21.05\ \text{kips}\ \therefore \text{OK}$$

Max. Bolt Bearing,

$$V_{\text{Design force per bearing surface}} = \frac{488\ \text{kips}}{36\ \text{bolts}} = 13.6\ \text{kips from pure shear} \downarrow$$

$$V_{\text{Design force per bearing surface}} = \frac{3172\ \text{kip-in} \times 4.5\ \text{in}}{2565\ \text{in}^2} = 5.6\ \text{kips max from moment} \downarrow$$

$$V_{\text{Design force per bearing surface}} = \frac{3172\ \text{kip-in} \times 12\ \text{in}}{2565\ \text{in}^2} = 14.8\ \text{kips max from moment} \rightarrow$$

$$V_{\text{Design Total force per bearing surface}} = \sqrt{(13.6 + 5.6)^2 + 14.8^2} = 24.2\ \text{kips max}$$

$$V_{\text{Allowable force per bearing surface}} = 0.9 \times L_c \times F_u \times 0.715" \quad (L_c = 1", F_u = 65\ \text{ksi})$$

$$58.5 \times 0.715" = 41.8\ \text{kips}\ \therefore \text{OK}$$

Flexure in Plates,

$$I_{\text{plates gross}} = \frac{1}{12}bh^3 = \frac{1}{12}\times 1\times\left(27\tfrac{1}{4}\right)^3 = 1686\ \text{in}^4$$

$$c_{\text{dis. extreme fiber}} = \frac{27\tfrac{1}{4}}{2} = 13\tfrac{5}{8}\ \text{in}$$

$$\sigma_{\text{Design stress}} = \frac{3172\times 13\tfrac{5}{8}}{1686} = 25.6\ \text{ksi}$$

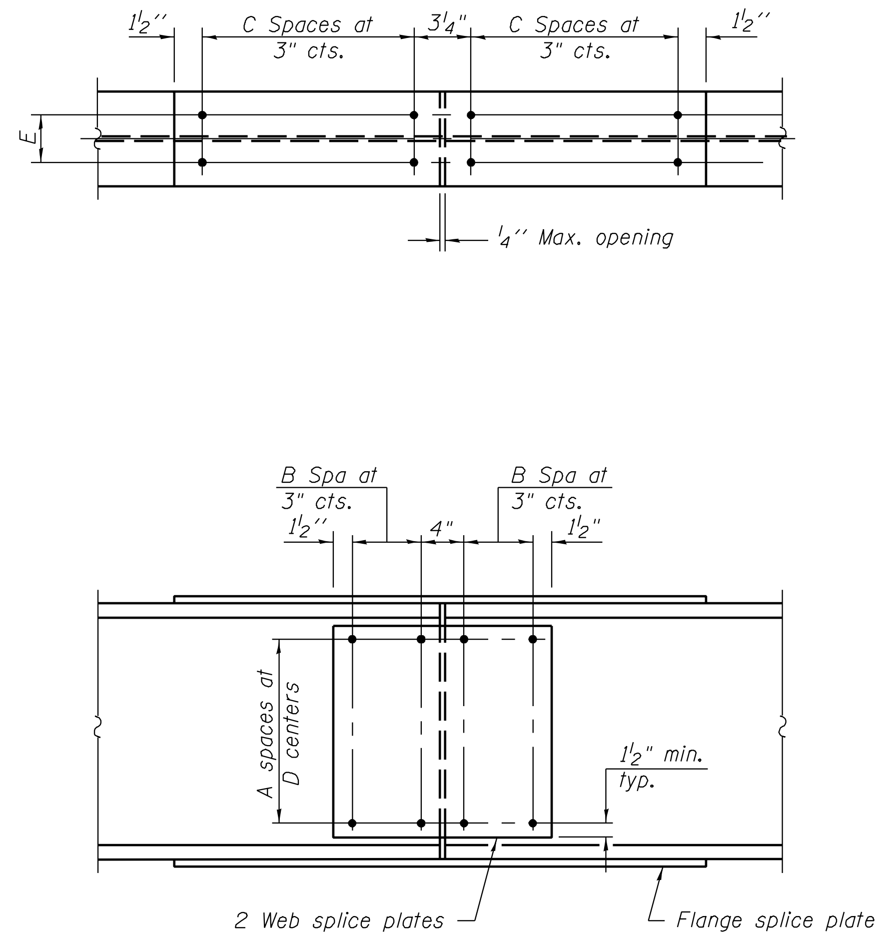
$$\sigma_{\text{Allowable stress}} = 50\ \text{ksi}\ \therefore \text{OK}$$

Shear in Plates,

$$V_{\text{Design force}} = 488\ \text{kips}$$

$$V_{\text{Allowable force}} = 0.58\times 50\times 27\tfrac{1}{4}\times 1 = 790\ \text{kips}\ \therefore \text{OK}$$

LRFD & LFD STANDARD BEAM SPLICES
FOR 75 PERCENT OF BEAM SECTION
BASED ON A.A.S.H.T.O. M270 GR36 & GR50



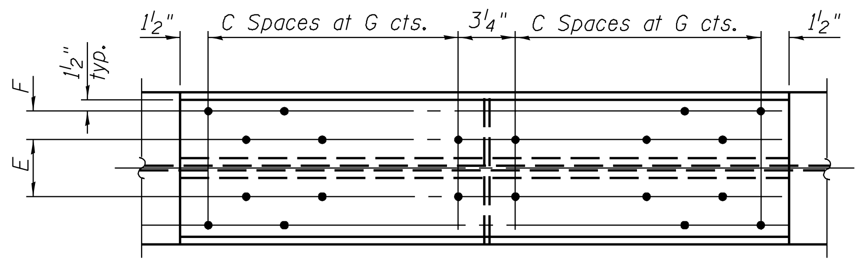
SPLICE WITH OUTSIDE FLANGE PLATE ONLY

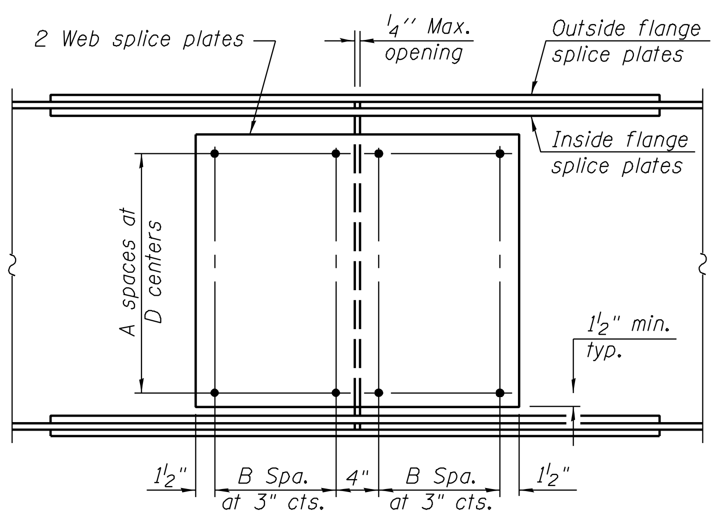
High strength bolts shall conform to AASHTO M-164 specification (ASTM A 325). Bolts $\frac{7}{8}"\ \phi$, open holes $\frac{15}{16}"\ \phi$.

STANDARD BEAM SPLICES

Figure 1

LRFD & LFD STANDARD BEAM SPLICES
FOR 75 PERCENT OF BEAM SECTION
BASED ON A.A.S.H.T.O. M270 GR36 & GR50





SPLICE WITH INSIDE AND OUTSIDE FLANGE PLATES

High strength bolts shall conform to AASHTO M-164 specification (ASTM A 325). Bolts $\frac{7}{8}$" ϕ, open holes $\frac{15}{16}$" ϕ.

STANDARD BEAM SPLICES

Figure 2

Steel	50 ksi

	Web Splice						Flange Splice										
	Plate Size						Outside Plate Size			Inside Plate Size							
Beam Size	Thickness (inches)	Length (inches)	Height (inches)	A	D (inches)	B	Thickness (inches)	Width (inches)	Length (inches)	Thickness (inches)	Width (inches)	Length (inches)	C	E (inches)	F (inches)	G (inches)	Weight (lbs)
W36X300	5/8	31	28 1/2	8	3 1/8	4	15/16	16 1/2	51 1/4	1	5 3/4	51 1/4	15	8	2 3/4	1 1/2	1292
W36X280	9/16	25	28 1/4	8	3 1/8	3	1	16 1/2	48 1/4	1 1/16	5 3/4	48 1/4	14	8	2 3/4	1 1/2	1184
W36X260	9/16	25	29	8	3 1/4	3	3/4	16 1/2	45 1/4	13/16	5 7/8	45 1/4	13	7 3/4	2 7/8	1 1/2	956
W36X245	1/2	25	29	8	3 1/4	3	11/16	16 1/2	42 1/4	3/4	5 7/8	42 1/4	12	7 3/4	2 7/8	1 1/2	843
W36X230	1/2	25	29	8	3 1/4	3	11/16	16 1/2	39 1/4	3/4	5 7/8	39 1/4	11	7 3/4	2 7/8	1 1/2	802
W36X210	1/2	25	31	9	3	3	1 3/8	12	120 1/4				19	8			1562
W36X194	1/2	25	31	9	3	3	1 5/16	12	108 1/4				17	8			1388
W36X182	7/16	19	31	9	3	2	1 3/16	12	102 1/4				16	7 1/2			1147
W36X170	7/16	19	31 1/8	9	3 1/8	2	1 1/8	12	96 1/4				15	7 1/2			1051
W36X160	7/16	19	31	9	3	2	1 1/16	12	90 1/4				14	7 1/2			958
W36X150	7/16	19	31 1/8	9	3 1/8	2	1	12	84 1/4				13	7 1/2			871
W36X135	7/16	19	31 1/8	9	3 1/8	2	13/16	12	72 1/4				11	7 1/2			682
W33X241	9/16	25	27	8	3	3	3/4	15 3/4	48 1/4	13/16	5 1/2	48 1/4	12	7 3/4	2 1/2	1 3/4	939
W33X221	1/2	25	27	8	3	3	11/16	15 3/4	44 3/4	3/4	5 1/2	44 3/4	11	7 3/4	2 1/2	1 3/4	824
W33X201	1/2	25	27 1/4	8	3	3	5/8	15 1/2	41 1/4	11/16	5 1/2	41 1/4	10	7 1/2	2 1/2	1 3/4	737
W33X152	7/16	13	28 1/2	8	3 1/8	1	1 1/16	11 1/2	90 1/4				14	7 1/2			856
W33X141	7/16	13	28 1/2	8	3 1/8	1	1	11 1/2	78 1/4				12	7 1/2			726
W33X130	7/16	13	28 1/2	8	3 1/8	1	7/8	11 1/2	72 1/4				11	7 1/2			619
W33X118	7/16	13	28 1/2	8	3 1/8	1	3/4	11 1/2	60 1/4				9	7 1/2			487
W30X211	1/2	25	24	7	3	3	11/16	15	53	3/4	5 1/8	53	11	7 3/4	2 1/8	2 1/8	853
W30X191	1/2	19	24	7	3	2	5/8	15	42 1/4	11/16	5 1/4	42 1/4	9	7 1/2	2 1/4	2	640
W30X173	7/16	25	24 1/4	7	3	3	9/16	15	38 1/4	5/8	5 1/4	38 1/4	8	7 1/2	2 1/4	2	595
W30X132	7/16	19	25 3/8	7	3 1/8	2	1 1/16	10 1/2	72 1/4				11	7 1/2			703
W30X124	7/16	13	25 3/8	7	3 1/8	1	15/16	10 1/2	72 1/4				11	7 1/2			597
W30X116	7/16	13	25 3/8	7	3 1/8	1	7/8	10 1/2	66 1/4				10	7 1/2			531
W30X108	7/16	13	25 3/8	7	3 1/8	1	13/16	10 1/2	60 1/4				9	7 1/2			470
W30X99	7/16	13	25 3/8	7	3 1/8	1	11/16	10 1/2	54 1/4				8	7 1/2			394
W27X178	1/2	25	21 1/4	6	3	3	5/8	14	51 1/4	11/16	4 3/4	51 1/4	9	7 1/2	1 3/4	2 1/2	716
W27X161	7/16	19	21 1/4	6	3	2	9/16	14	46 1/4	5/8	4 3/4	46 1/4	8	7 1/2	1 3/4	2 1/2	563
W27X146	7/16	19	21 1/2	6	3	2	1/2	14	39 1/2	9/16	4 7/8	39 1/2	7	7 1/4	1 7/8	2 3/8	474
W24X162	1/2	25	19 3/4	5	3 1/4	3	1 1/4	13	114 1/4				18	7 1/2			1375
W24X146	7/16	25	19 3/4	5	3 1/4	3	1 1/8	12 3/4	102 1/4				16	7 3/4			1119
W24X131	7/16	19	19 3/4	5	3 1/4	2	1	12 3/4	90 1/4				14	7 1/4			884
W24X117	7/16	25	19 3/4	5	3 1/4	3	7/8	12 3/4	78 1/4				12	7 1/4			751
W24X104	7/16	19	19 3/4	5	3 1/4	2	13/16	12 3/4	72 1/4				11	7 1/4			633
W21X132	9/16	31	16 3/4	4	3 3/8	4	1 1/16	12 1/4	90 1/4				14	7 3/4			983
W21X122	1/2	25	17	4	3 1/2	3	1	12 1/4	84 1/4				13	7 3/4			841
W21X111	7/16	25	17	4	3 1/2	3	15/16	12 1/4	78 1/4				12	7 1/4			742
W21X101	7/16	25	17	4	3 1/2	3	13/16	12 1/4	72 1/4				11	7 1/4			632

Table 1

Steel 36 ksi

	Web Splice						Flange Splice										
	Plate Size						Outside Plate Size			Inside Plate Size							
Beam Size	Thickness (inches)	Length (inches)	Height (inches)	A	D (inches)	B	Thickness (inches)	Width (inches)	Length (inches)	Thickness (inches)	Width (inches)	Length (inches)	C	E (inches)	F (inches)	G (inches)	Weight (lbs)
W36X300	5/8	19	28 1/2	8	3 1/8	2	15/16	16 1/2	39 1/4	1	5 3/4	39 1/4	11	8	2 3/4	1 1/2	927
W36X280	9/16	19	28 1/4	8	3 1/8	2	1	16 1/2	39 1/4	1 1/16	5 3/4	39 1/4	11	8	2 3/4	1 1/2	946
W36X260	9/16	19	29	8	3 1/4	2	3/4	16 1/2	36 1/4	13/16	5 7/8	36 1/4	10	7 3/4	2 7/8	1 1/2	752
W36X245	1/2	19	29	8	3 1/4	2	11/16	16 1/2	33 1/4	3/4	5 7/8	33 1/4	9	7 3/4	2 7/8	1 1/2	654
W36X230	1/2	13	29	8	3 1/4	1	11/16	16 1/2	30 1/4	3/4	5 7/8	30 1/4	8	7 3/4	2 7/8	1 1/2	548
W36X210	1/2	13	31	9	3	1	1 3/8	12	90 1/4				14	8			1105
W36X194	1/2	13	31	9	3	1	1 5/16	12	84 1/4				13	8			1004
W36X182	7/16	13	31	9	3	1	1 3/16	12	78 1/4				12	7 1/2			861
W36X170	7/16	13	31 1/8	9	3 1/8	1	1 1/8	12	72 1/4				11	7 1/2			775
W36X160	7/16	13	31	9	3	1	1 1/16	12	66 1/4				10	7 1/2			692
W36X150	7/16	13	31 1/8	9	3 1/8	1	1	12	60 1/4				9	7 1/2			616
W36X135	7/16	13	31 1/8	9	3 1/8	1	13/16	12	54 1/4				8	7 1/2			498
W33X241	9/16	19	27	8	3	2	3/4	15 3/4	37 3/4	13/16	5 1/2	37 3/4	9	7 3/4	2 1/2	1 3/4	727
W33X221	1/2	13	27	8	3	1	11/16	15 3/4	34 1/4	3/4	5 1/2	34 1/4	8	7 3/4	2 1/2	1 3/4	566
W33X201	1/2	13	27 1/4	8	3	1	5/8	15 1/2	30 3/4	11/16	5 1/2	30 3/4	7	7 1/2	2 1/2	1 3/4	490
W33X152	7/16	13	28 1/2	8	3 1/8	1	1 1/16	11 1/2	66 1/4				10	7 1/2			661
W33X141	7/16	13	28 1/2	8	3 1/8	1	1	11 1/2	60 1/4				9	7 1/2			587
W33X130	7/16	13	28 1/2	8	3 1/8	1	7/8	11 1/2	54 1/4				8	7 1/2			496
W33X118	7/16	13	28 1/2	8	3 1/8	1	3/4	11 1/2	48 1/4				7	7 1/2			414
W30X211	1/2	19	24	7	3	2	11/16	15	40 1/4	3/4	5 1/8	40 1/4	8	7 3/4	2 1/8	2 1/8	647
W30X191	1/2	13	24	7	3	1	5/8	15	34 1/4	11/16	5 1/4	34 1/4	7	7 1/2	2 1/4	2	496
W30X173	7/16	13	24 1/4	7	3	1	9/16	15	30 1/4	5/8	5 1/4	30 1/4	6	7 1/2	2 1/4	2	413
W30X132	7/16	13	25 3/8	7	3 1/8	1	1 1/16	10 1/2	54 1/4				8	7 1/2			516
W30X124	7/16	13	25 3/8	7	3 1/8	1	15/16	10 1/2	54 1/4				8	7 1/2			475
W30X116	7/16	13	25 3/8	7	3 1/8	1	7/8	10 1/2	48 1/4				7	7 1/2			416
W30X108	7/16	13	25 3/8	7	3 1/8	1	13/16	10 1/2	42 1/4				6	7 1/2			362
W30X99	7/16	13	25 3/8	7	3 1/8	1	11/16	10 1/2	42 1/4				6	7 1/2			331
W27X178	1/2	19	21 1/4	6	3	2	5/8	14	41 1/4	11/16	4 3/4	41 1/4	7	7 1/2	1 3/4	2 1/2	567
W27X161	7/16	13	21 1/4	6	3	1	9/16	14	36 1/4	5/8	4 3/4	36 1/4	6	7 1/2	1 3/4	2 1/2	427
W27X146	7/16	13	21 1/2	6	3	1	1/2	14	30	9/16	4 7/8	30	5	7 1/4	1 7/8	2 3/8	348
W24X162	1/2	19	19 3/4	5	3 1/4	2	1 1/4	13	84 1/4				13	7 1/2			1018
W24X146	7/16	19	19 3/4	5	3 1/4	2	1 1/8	12 3/4	78 1/4				12	7 3/4			855
W24X131	7/16	13	19 3/4	5	3 1/4	1	1	12 3/4	66 1/4				10	7 1/4			642
W24X117	7/16	13	19 3/4	5	3 1/4	1	7/8	12 3/4	60 1/4				9	7 1/4			536
W24X104	7/16	13	19 3/4	5	3 1/4	1	13/16	12 3/4	54 1/4				8	7 1/4			466
W21X132	9/16	19	16 3/4	4	3 3/8	2	1 1/16	12 1/4	66 1/4				10	7 3/4			696
W21X122	1/2	19	17	4	3 1/2	2	1	12 1/4	66 1/4				10	7 3/4			657
W21X111	7/16	19	17	4	3 1/2	2	15/16	12 1/4	60 1/4				9	7 1/4			569
W21X101	7/16	13	17	4	3 1/2	1	13/16	12 1/4	54 1/4				8	7 1/4			441

Table 2